PHYSICAL MODELLING AND NUMERICAL ANALYSIS OF VERTICAL PILES

Synopsis of the PhD Thesis

Submitted by

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1. ABSTRACT

Experimental studies were conducted to evaluate the performance of socketed piles under compressive load. The details of load transmission via friction on model piles socketed in pseudo-rock samples are investigated. Indigenously designed and fabricated loading frame, a number of socketed model pile load tests were performed. In the studies, two aluminium piles with outside diameters of 60 and 80 mm were utilized. By varying the length of the piles, several length-to-diameter ratios were examined. To prevent the significance of end resistance and to generate skin frictional roughness, sponge was placed at the bottom of the pile. By altering the quantities of cement and plaster of Paris, several combinations of pseudo-rock specimens were developed. UCS experiments were performed on samples to assess the influence of strength on skin friction. Shear characteristics of pseudo-rock samples were also calculated. Research results demonstrate that the load-displacement curve is nonlinear. An analytical connection is developed with pseudo-rocks compressive strength and highest skin friction generated in pile. A strong connection is shown between the laboratory findings and the findings attained by applying empirical equations recommended by other researchers. Also, pseudo-rock, sand bed and piles were numerically modeled using PLAXIS 3D. Triaxial tests on sand samples were used to determine soil properties, whereas laboratory tests were used to estimate pile parameters. By adding interface elements across the soil and the pile, the influence of soil-pile interaction has been assessed. A several parametric investigations are conducted by modifying the pile arrangements, and the system output are compared with experimental results.

Keywords: Pile foundation; axial load; settlement; skin friction; pile load test; FEM.

2. INTRODUCTION

Now a days, large diameter piles are utilized to support massive load from superstructures in recent construction developments. Bored piles with diameters of 1000 mm to 1500 mm are employed in almost all construction projects. Because these piles are designed to carry enormous weights (ranging from 450 to 650 tons per pile), they must be lowered to bedrock and drilled in the rock. this is often stated as a termination criterion in piling specifications. Where the piles are standing on rock at low height or on sloping rock, socketing in rock remains extremely important for all practical reasons. Anywhere the hard rock is located at deeper levels, has a greater overburden to oppose adjacent deformation, usually piling requirements do not need pile socketing, and excessive chiseling for specified socket length is a waste of time, and massive chiseling may sometimes degrade the superiority of hard rock. Except for

underground utility lines and boulders, boring up to the rock surface should be rather easy. Socketing in rock will provide a variety of practical concerns, including from rock categorization to socketing length and final termination requirements.

Till recent time, it was common to adopt allowable bearing pressure of 3.0 MPa for sound rocks like Basalts, and 2.5 MPa for weaker rocks like volcanic Breccia and Tuff. During pile installation, chiseling energy is still being used for pile termination in weathered rocks. These practices appear to be very conservative as they neglect, low values of the side friction between the pile and socket interface. Moreover, the additional capacity arising due to frictional resistance between the shaft and the weathered strata is not accounted in the existing designs. To understand how a socketed pile would behave in the field, in-situ pile load tests were performed. During these tests, the pile was subjected different types of loads. Due to time and economic constraints, testing cannot be performed upon all piles at a construction site due to significant number of piles associated in it. Furthermore, in some conditions, such as congested and heavily populated urban regions, the static load test is unsustainable. The Indian code, specifies one or more preliminary pile load test, to be performed for important or major projects. It also specifies routine pile load tests on working piles, to be conducted regularly at one-half to two percent, which may be increases based on nature, type of structure and the strata situations. This frequency, which obviously considers the practical constraints involved in field tests, may not give sufficient idea to judge the performance of remaining piles founded under different sub-soil conditions. As a result of the practical limitations and cost concerns, the investigation for alternate approaches for assessing pile load-deformation behavior has increased. The accessibility early knowledge on the load-displacement performance of socketed piles would substantially improve on-site decision-making and boost trust in the methods used. However, these have not been frequently applied to the field situations, since they rely on the parameters and properties that are either difficult to determine or are not a part of the routine geotechnical investigations. Load displacement analysis using elastic theories are also available. At best, yield solutions at lower values of working loads, where the behavior of the rock sockets are expected to be elastic. In contrast, according to literature research and field observations, the load-displacement response of rock socket is non-linear, making elastic theories difficult to give a satisfactory fit, especially at higher loads. There are empirical/semi-empirical approaches for determining pile side friction and tip resistance value of socket. Hence the possibility of applying these empirical approaches for socketed pile in this region needs to be studied.

Breaking the sound rock for a minimum of 1time diameter (1D) is the current practice of socketed piles in rock. When the diameter of pile is greater than 1.2 m, then the socketed length will be decided by chiseling energy criteria. these factors have the potential to damage the rock's structure. To achieve better contact in between base of the pile and the rock, the pile must be socketed by breaking through weathered, soft and hard rock to an acceptable length. There are usually different types of rock. First and foremost, appropriate rock layer categorization is required. Field test data such as Rock Quality Designation (RQD), Core Recovery (CR), and Standard Penetration Test (SPT) findings, as well as the crushing strength of rock core samples, can be used to this purpose. Soil investigations are required under every pier position for big structures such as bridges and flyovers. As a result, the above-mentioned tests must be performed at each pier location.

These studies present calculation of skin friction and behavior of socketed pile in sand / rock under axial loading. Also, measurement of axial load using Finite element method (FEM) methods.

2.1 RESEARCH GAP

Other researchers have carried out pseudo-rock testing by using different chemicals with cement and a small number of specimens are tested. Also, they have used rocks having lower strengths (UCS values of 0.9-17 MPa).

3. OBJECTIVES

I. To estimate skin friction and socketing length of piles in sand / rock strata.
II. To study the behavior of socketed piles by conducting laboratory model tests on pile.
III. To validate laboratory results using numerical modeling (PLAXIS 3D).

4. LITERATURE REVIEW

The pile foundation among the most ancient foundation techniques, and it has a long history of usage as a structural element. Usually, piles are designed to transfer loads to stronger strata and to avoid weak layer. [1] has reported economy, construction period, labour, concrete, steel and energy, as a principle reason of using a socketed pile. Theoretically, in socketed piles, loads introduced on pile top which transmitted to the surrounding strata through both tip and skin resistance. The relative significance of all of these aspects dependent shape of the pile, socket, and on rigidity of the pile with reference to the adjacent rock, and the rock underlying it [2]. Until recently, little reliance had been placed on sidewall shear as a load supporting component; instead, these foundations have been designed to support loads by end bearing only. The

mechanism of sidewall shear development in rock is very complex, and includes both adhesion and friction [3-6].

[7] reviewed some of the methods of socket design. According to them, four basic methods are adopted. These may be based exclusively on end bearing criterion, concrete-rock bond, based on a permissible end resistance and carrying the residual load through bond or based on predicted end bearing. In their design review they recommended a variety of bond values for piles socketed in rock.

[8] studied that, "for usual embedment ratios of the sockets (Lsoc/D between 2 to 4), the end bearing carried only a small division (5 to 25%) of the total imposed load". His presented charts giving the values of rock bond is result of the UCS of rock specimen. [9] in his discussion, advocated the recommendations of [8], while stating that side wall shear resistance essentially behaved in a non-brittle way, it didn't reduce even after the pile-rock bond was broken. Studies on long term observation on a socketed pier indicated that, the end bearing pressure in the socket improved with time, though at a falling rate. However, even after four years, value of the end bearing pressure was found to be far less than the design value. The author gave a method for design of rock socketed piles by using the load distribution curves proposed by [10-14] and used limiting bond strength as a criterion for socket geometry. This method, however, did not give the settlement under a given pile load.

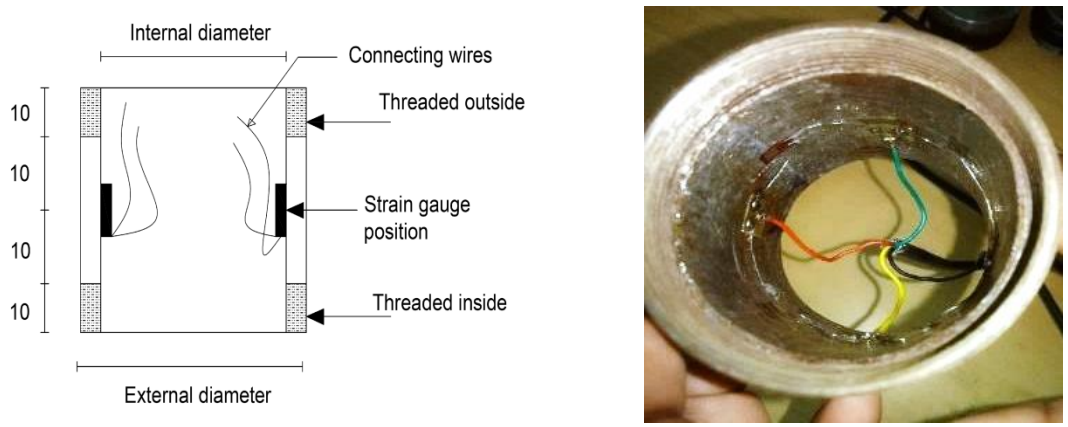
5. CONTRIBUTION

5.1 EXPERIMENTAL STUDIES ON ROCK SOCKETED PILE

Instrumented pile load tests are among the suggested laboratory investigations, with the objective of attempting to help additional knowledge on load transmission mechanisms and skin resistance mobilization in piles. In the studies, "an aluminum hollow pipe is utilized as a model pile. The hallow pipe is split into pieces to simulate the model piles required socketed length-to-diameter (Ls/D) ratio. The investigations used two diameters (external) of piles, 60 mm and 80 mm, with a pile tube thickness of 5 mm. The piles socket lengths (Ls) ranged from two times the diameter (2D) to seven times the diameter (7D). Strain gauges (SG) of type FLA-3-350-11 were installed, and these hollow pipes were cut at fix spacing to them. The strain gauges in this model pile were fixed with CN adhesive [65]". Fig. 1 show a schematic representation using the strain gauge and the modelled pile configuration. On the inside surface of the aluminum model pile, strain gauges were installed. The model pile was split into different pieces to allow for this layout. Each section had a threaded connection, when combined

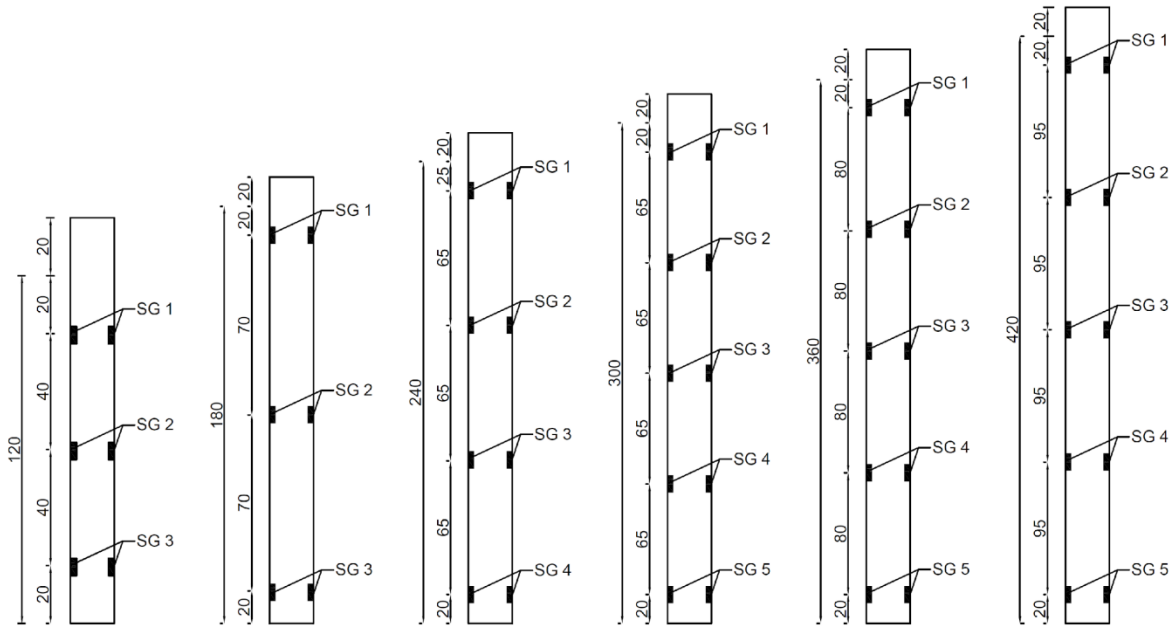
together created a single pile unit. The height of every piece was 50 mm. The strain gauges were glued into place in an aluminum pipe and then covered in a water-resistant adhesive. “The test tank is made of 6 mm thick mild steel plate with an interior diameter of 400 mm and a height of 1000 mm”. To hold two linear variable displacement transducers (LVDT) for measuring settlement data on the pile top, a metal ring with flanges was connected. It is possible to determine the mean settlement received from the two LVDTs. The pile movement was measured using an LVDT while the axial force was transferred. The highest measurement limit of the LVDT device is 25 mm, with a minimum limit of 0.25 mm.

Experimental set-up is shown in Fig. 2 and 3 respectively. A hydraulic jack “connected to the bottom of the loading frame enabled the transmission of compressive force on the pile head in the experimental set. The hydraulic jack was fitted with a load-cell. The settlement of the pile top was measured using two LVDTs mounted on wing plates connected to symmetrically opposite sides of the pile. To analysed the phenomena of pile load transfer only due to side shear and end bearing has to be completely removed. The occurrence of compressible muck or clay seams directly beneath the pile foundation causes this condition in the field.



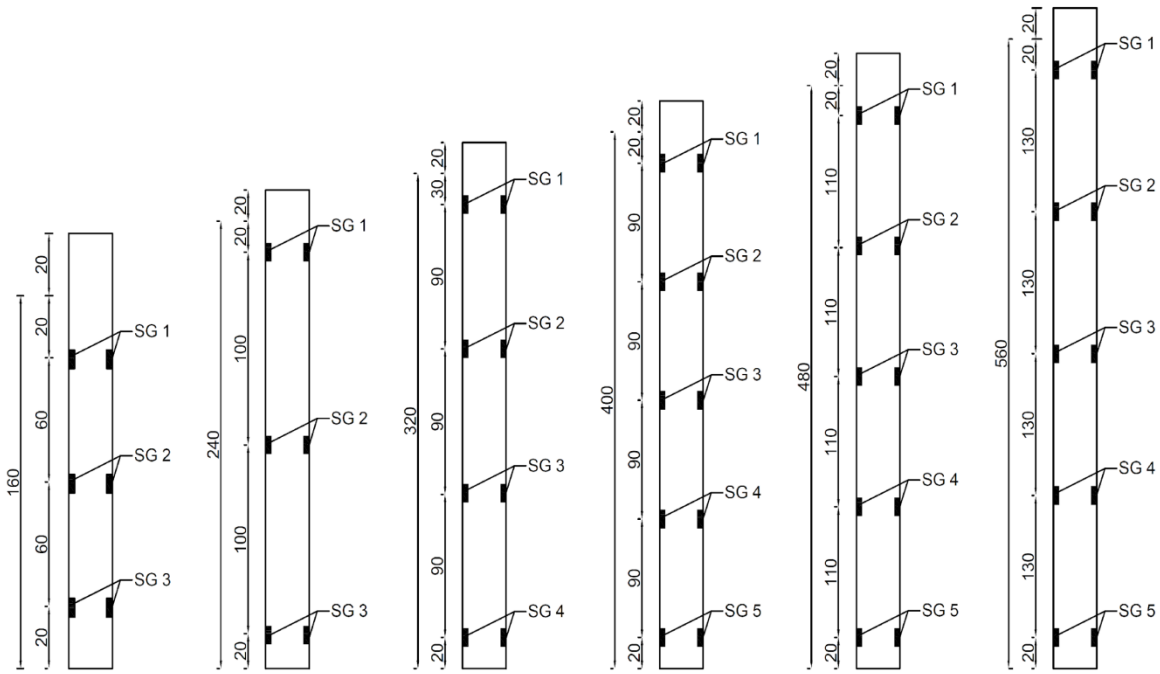
(a) **(b)**

Fig. 1 (a) Schematic view of typical model pile segment, (b) arrangement of strain gauges in the pile. (all dimensions are in mm)



Ls/D=2 Ls/D=3 Ls/D=4 Ls/D=5 Ls/D=6 Ls/D=7

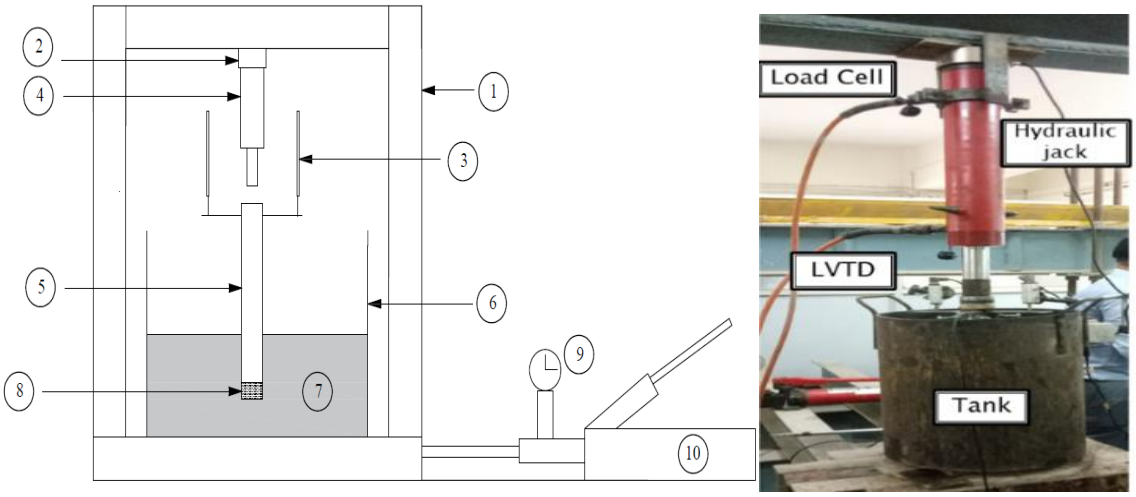
(a) Piles diameter = 60 mm.



Ls/D=2 Ls/D=3 Ls/D=4 Ls/D=5 Ls/D=6 Ls/D=7

b) Piles diameter = 80 mm.

Fig. 2. Strain gauges location on piles. (all dimensions are in mm)



(a) (b)

Fig. 3 Experimental set-up (a) Schematic view of experimental setup (1= loading frame, 2= Load cell, 3= LVDT, 4= hydraulic jack, 5= model pile, 6 = circular tank, 7 = pseudo-rock, 8 = Styrofoam, 9 = pressure indicator, 10 =hydraulic pump) (b) A set-up of socketed pile load test.

5.2 RESULTS AND DISCUSSION

For settlement measurements, the mean of the LVDT data was recorded. Figure 4 depicts a study of the load-displacement curves for P_1 (D = 60 mm) in various pseudo-rock mixtures (M_1 to M_{11}). As indicated in Fig. 4, the highest loads corresponded to settlements ranging from 1.0 mm to 1.75 mm. These charts were created using Ls/D ratio of two and the highest measured load. "The skin friction loads observed were 49.8 kN, 44.2 kN, 38.1 kN, 28.6 kN, 25.4 kN, 19.5 kN, 13.2 kN, 6.1 kN, 3.2 kN, 2.5 kN, 1.2 kN in pseudo-rock mixes M_1 to M_{11} respectively, and the corresponding settlements were found to be 1.75 mm, 1.50 mm, 1.50 mm, 1.75 mm, 1.25 mm, 1.50 mm, 1.50 mm, 1.25 mm, 1 mm, 1 mm, 1 mm respectively. The load bearing ability of pile improved along with the UCS of pseudo-rock. Using mix M_2, the load-settlement behavior of the various diameter piles P_1 and P_2 (D = 80 mm) is also examined. Figure 5 analyzes the loads-settlements for piles P_1 and P_2. The maximum loads that P_1 and P_2 can support are 49.3 kN and 90.2 kN, respectively. As illustrated in Fig. 5, the corresponding settlements of 1.75 mm and 2 mm were also noticed. These findings confirm the fact that piles load bearing capability improved along with its diameter. Additionally, it is observed that if a piles diameter is raised by around 33 %, its load-bearing capacity rises by around 82.96 %".

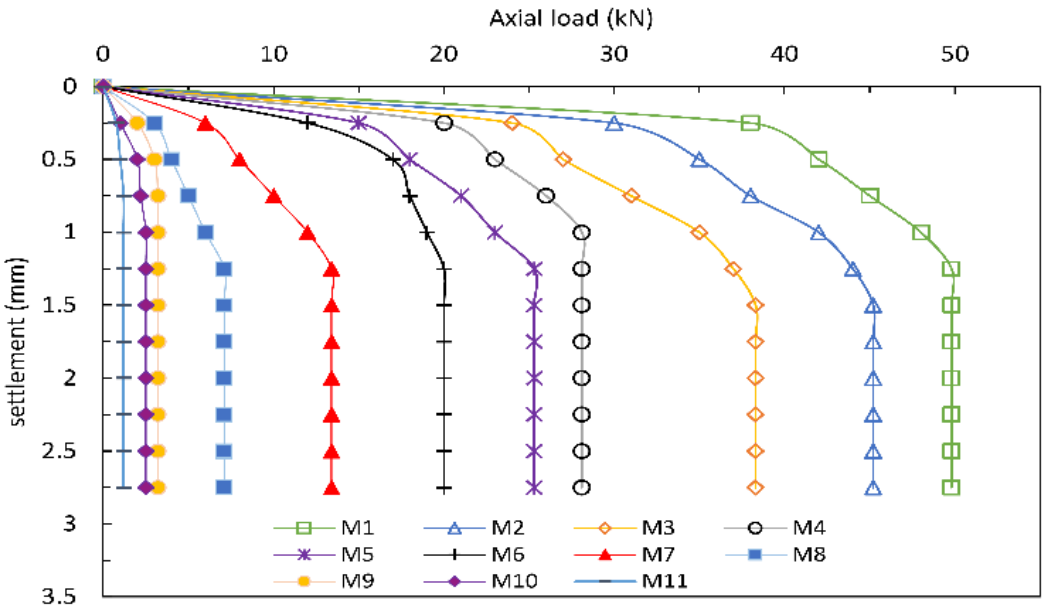


Fig. 4 Load - settlement curve for P_1 pile in Ls/D=2.

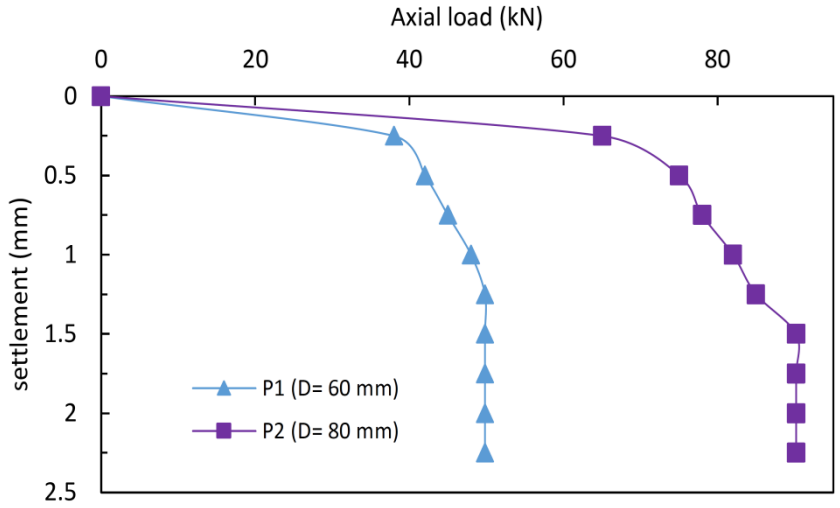


Fig. 5 Load-settlement curve for (Ls/D =2) socketed piles in M_2 mix.

5.3 PILE LOAD DISTRIBUTION

The skin frictional forces on the piles sides are calculated using the observed stresses obtained from strain gauge measurements. As a result, equation (1) may be used to determine "the skin frictional load (Q) in the pile.

$$Q=\varepsilon EA \quad (1)$$

Where, ε corresponds to measured strain, E is the modulus of elasticity of model pile and A is the cross-sectional area of the pile. Measured strain values were used to study the behavior of loads were transmitted. Using equation (1), skin frictional values were computed at different levels on the pile. As compressive loads were provided at the head of piles P_1 and P_2, respectively, Fig. 6 show the variations of frictional forces mobilized throughout the height (depth) of piles with Ls/D ratios of 2, 4, and 6. Applied and maintained steady incremental loads of 10 kN, 20 kN, etc., and Fig. 6 illustrates how these frictional forces change with pile length.

5.4 CALCULATION OF UNIT SKIN FRICTION

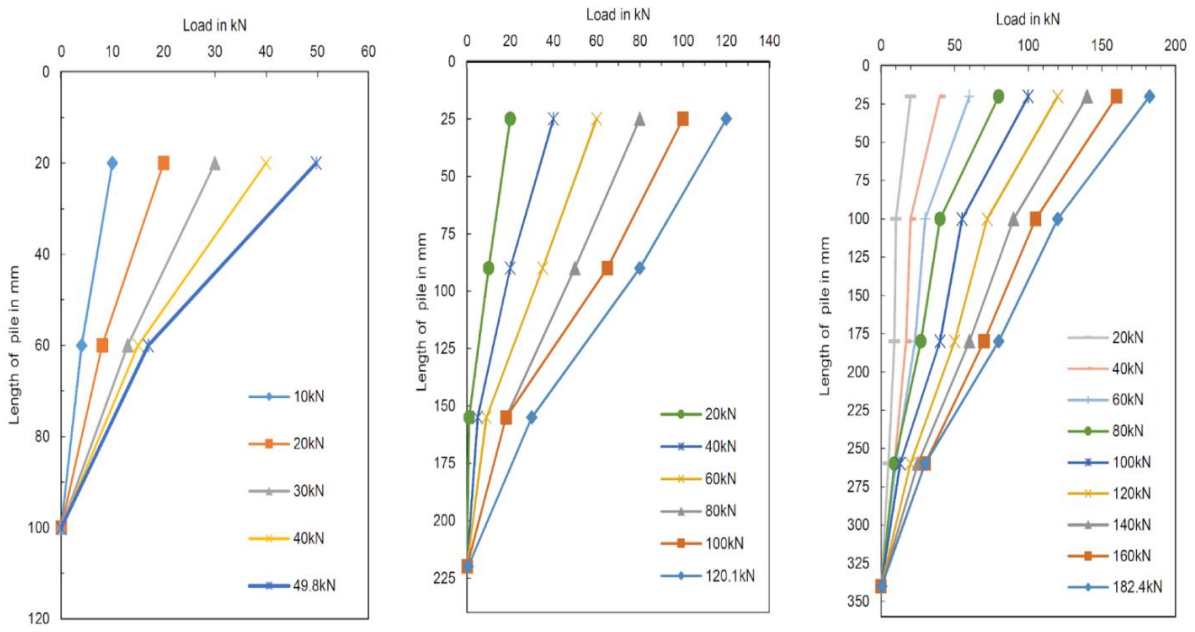
"For the purpose of calculating unit skin friction, the frictional loads were estimated at various strain gauge locations. The amount of skin friction (f) mobilized between the same two levels of strain gauges is equal to the difference between the skin frictional forces divided by the side surface area of the pile between consecutive strain gauges. Unit skin friction, or f, could be estimated using equation (2) for a small portion of the pile length.

$$f=\frac{\Delta Q}{\pi DL} \quad . \quad (2)$$

where D is the piles diameter, L is the space between subsequent strain gauges, and ΔQ is the difference in axial load measured at each gauges subsequent location. At the halfway point of subsequent strain gauges, this unit skin friction mobilized is plotted. In Figs. 7 (a) and 7 (b), respectively, the variation of unit skin friction along the length of piles P_1 and P_2 with Ls/D ratios of 2, 4, and 6 is depicted. In general, it is discovered that the mobilized skin friction increases with increasing axial force and does so until the failure loads. Utilizing the axial loads measured at the locations of the strain gauges, the mobilized skin friction was estimated. The midpoint of each subsequent position for the strain gauge is shown against these loads. For Pile P_1 (Ls/D = 2), Fig. 7 (a) shows that the skin friction is higher at the top and lower at the bottom.

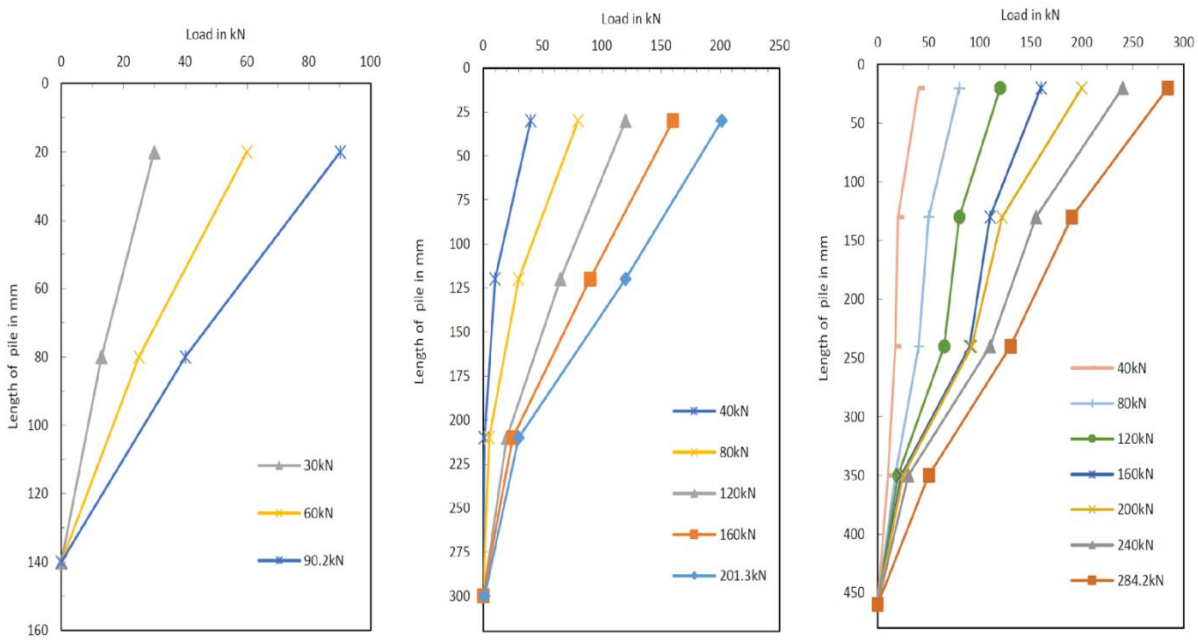
According to Fig. 7, the head of the pile has the most skin friction created over a length of 60 mm to 130 mm measured from the top. The skin friction is observed to drop and then increase again close to the base of the pile when Ls/D is 6 or higher, creating an S-shaped variation with pile depth. When Ls/D ratio is 6 or more, two peaks of skin friction are seen, one at the top of the pile and the other close to the bottom (80 to 120 mm from the base of the

pile). Near the top compared to the base, a higher rate of skin friction is shown to be mobilized. By using finite element analysis, identical results were found by [103] and [94].



Ls/D = 2 Ls/D = 4 Ls/D = 6

(a) Piles diameter = 60 mm and mix M_2.



Ls/D = 2 Ls/D = 4 Ls/D = 6

(b) Piles diameter = 80 mm and mix M_2.

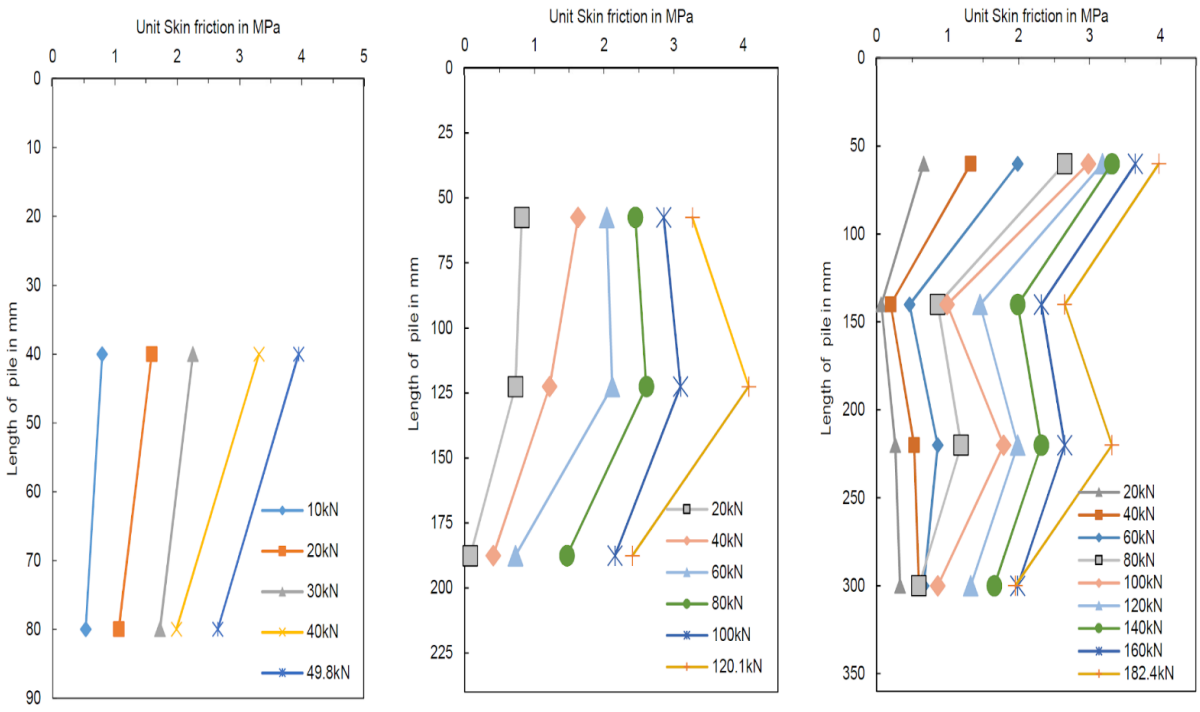
Fig. 6 frictional load distribution curves for different Ls/D and diameters of piles.

Skin friction at the pile tip decreases when end bearing resistance starts to contribute; yet, due to sponge compression, there is significant pile movement. This is in line with [95] and [104] observed. Only around 10 to 20 % of the axial force is transferred to the bottom of the pile, according to [22] and [15]. When the model pile's diameter was 80 mm and it had Ls/d ratios of 2, 4, and 6, similar skin friction fluctuation behavior was seen in Fig. 7 (b)" [65]. The UCS values of pseudo-rock compositions are found to raise the ultimate skin resistance mobilized at the head of the pile. As a result, the suggested empirical relationship between f_{max} and qu is established and is provided in Eq (3). This equation is appropriate for pseudo-rock with a UCS of up to 48 MPa.

$$f_{max} = 0.478(q_u)^{0.44} \quad (3)$$

"Where f_{max} is the maximum skin friction mobilized at the top of pile (MPa) and q_u is the UCS value of the pseudo-rock (MPa). The values of f_{max} obtained using proposed equation (3) are compared to the values produced using the equations suggested by other researchers in Fig. 8. The results acquired using the newly proposed equation (3) is examined to the estimates of maximum skin friction made using the five empirical relations suggested by previous researchers in this study. When compared to the findings derived using the suggestions of [3]

and [105], the results as generated using equation (3) overestimate the skin friction values up to 50%".



Ls/D = 2 **Ls/D = 4** **Ls/D = 6**

Fig. 7 Frictional load distribution curves for different Ls/D and diameters of piles.

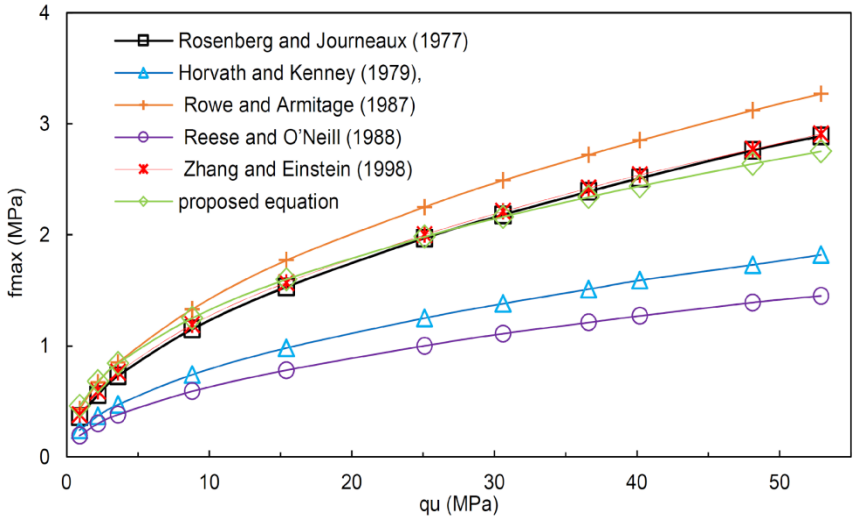


Fig. 8 Comparison of unit skin friction values

5.5 EXPERIMENTAL STUDIES ON SKIN FRICTION OF PILE IN SAND

When the top soil surface is insufficient to offer adequate shear strength to retain the superstructure, a piles foundation is typically implemented to distribute the weight to a deeper level. Mobilized skin resistance and the pile base friction will operate combine to oppose the imposed load. The bored pile is among the highest established and commonly utilized methods, and it may be utilized in a number of ground circumstances, along with soils and rocks. Bore holes, on the other hand, must be thoroughly cleaned to guarantee that there are no particles or drilling muck at the pile tip. When loose sediments present at the pile base, a pile under loading may undergo some settlement until the end bearing capacity is deployed**.** [106-107] discovered the pile load - carrying problem as a result of surrounding and bottom debris**.**

A number of research investigations are provided in this analysis. In sandy soil, experimental studies are carried out on model aluminum pile without and with sponge at the pile base. The deployed load is kept constant to record the entire load-displacement performance of piles.

5.6 EXPERIMENTAL SETUP

In an experiment arrangement that was created particularly for this, the model laboratory experiments were conducted. According to [102], the entire process of compacting the sand material was controlled in order to achieve a consistent density and to eliminate the enclosed voids. The compaction was done in stages, with each layer receiving 50 strokes of the conventional tamping bar and a thickness of 50 mm. “A steel bar with a 16 mm diameter, 0.60 m in length, and a bullet-pointed lower end serves as the tamping bar”. The cross-section of the tank received an even distribution of the bars strokes. to get a 65 % or higher average relative density.

5.7 RESULTS AND DISCUSSION

Figures 9 and 10 demonstrates the load-settlement graphs for piles with and without sponge at the pile tip. The graph for the typical pile exhibits a linear trend during the initial phases of this process and a nonlinear expanded trend of settlements with loading during the latter phases. When a weight is transmitted to a sponge pile, the pile may rapidly settle if the imposed force is greater than the skin friction because of the muck present. For the purpose of calculating vertical settlement, the average of the LVDT measurements was taken. In Figures 9 and 10, the load-displacement curves for different Ls/D are compared. As indicated in Fig. 9, the equivalent settlements to the measured loads of 3.5 kN and 5.3 kN were determined to

be 15 mm. 4.8 and 6.4 kN of measured loads and 15 mm of corresponding settlements are shown in Fig. 10. The axial pile capacity has been seen to rise together with the Ls/D ratio.

Recorded strain values were used to study the behavior of transferred load. Using equation (5.1), skin frictional forces were computed at different levels on the pile. The deviation of the frictional forces generated throughout the length of piles with Ls/D ratio of 4 is shown in Figure11, where axial loads were imposed at the head of the pile. These numbers show that as Ls/D ratios improve, pile load carrying capacities rises as well. The ultimate skin frictional loads were highest at the top of piles, and they gradually decreased along the depth of the pile before becoming lowest there. When taking into account the vertical equilibrium of the pile, it is discovered that the axial applied load is almost exactly equal to the skin frictional forces." [65].

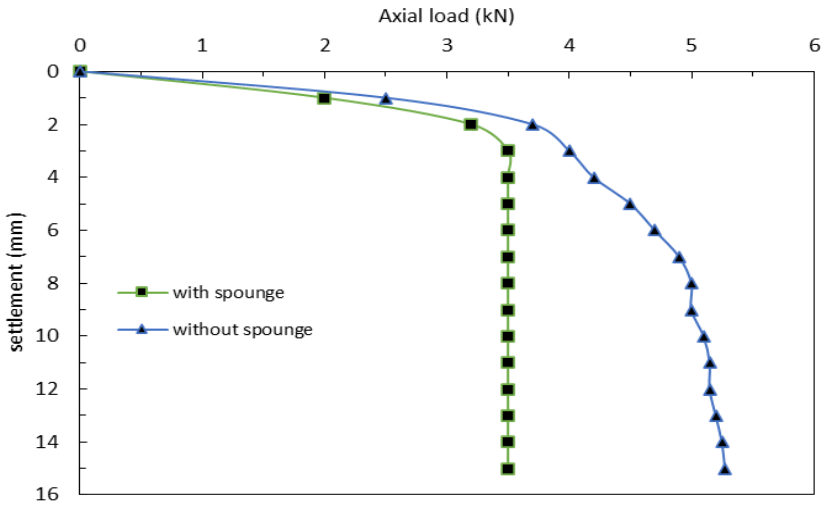


Fig 9. Load-settlement curve for pile D=60mm, Ls/D=4

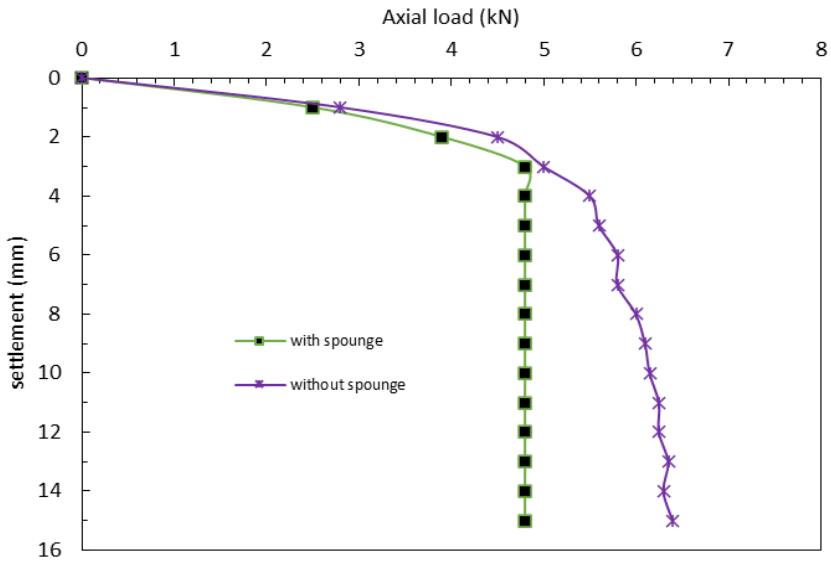
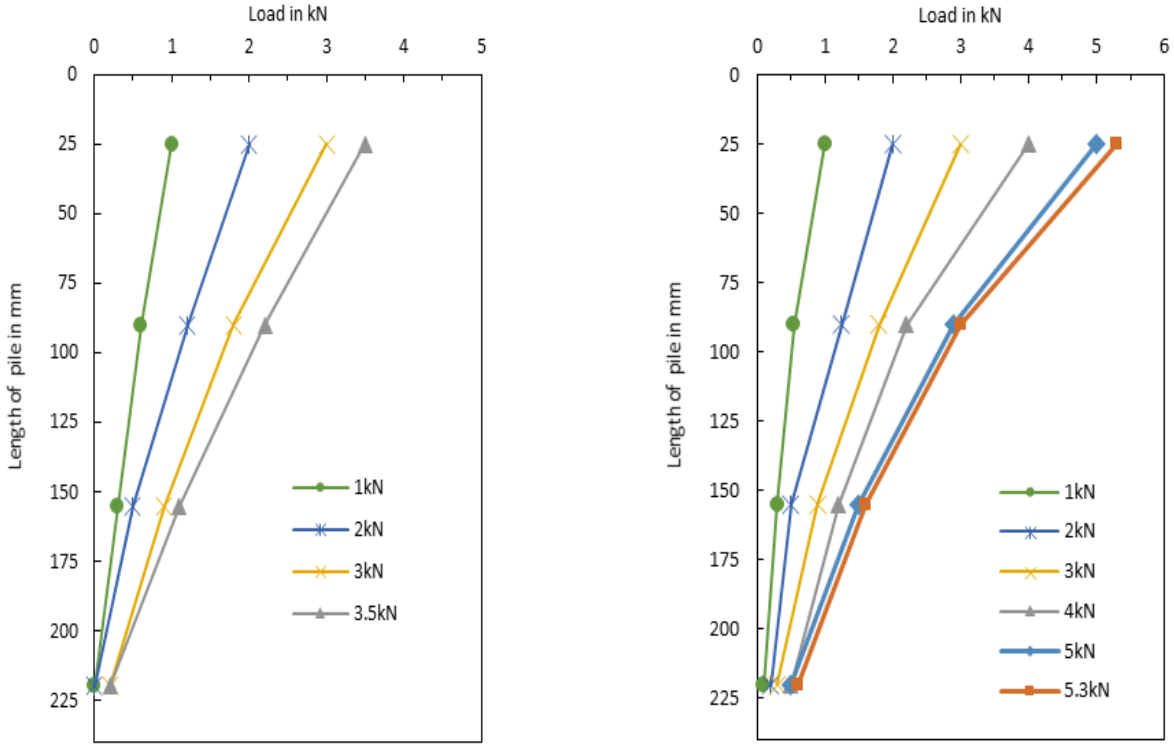


Fig 10. Load-settlement curve for pile D=60mm, Ls/D=6

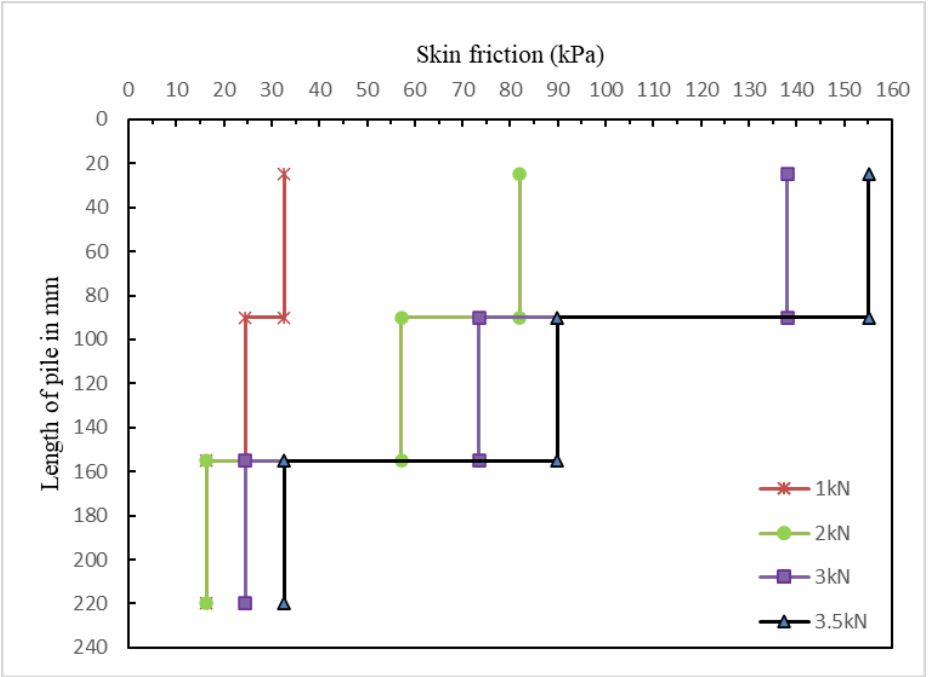


(b)

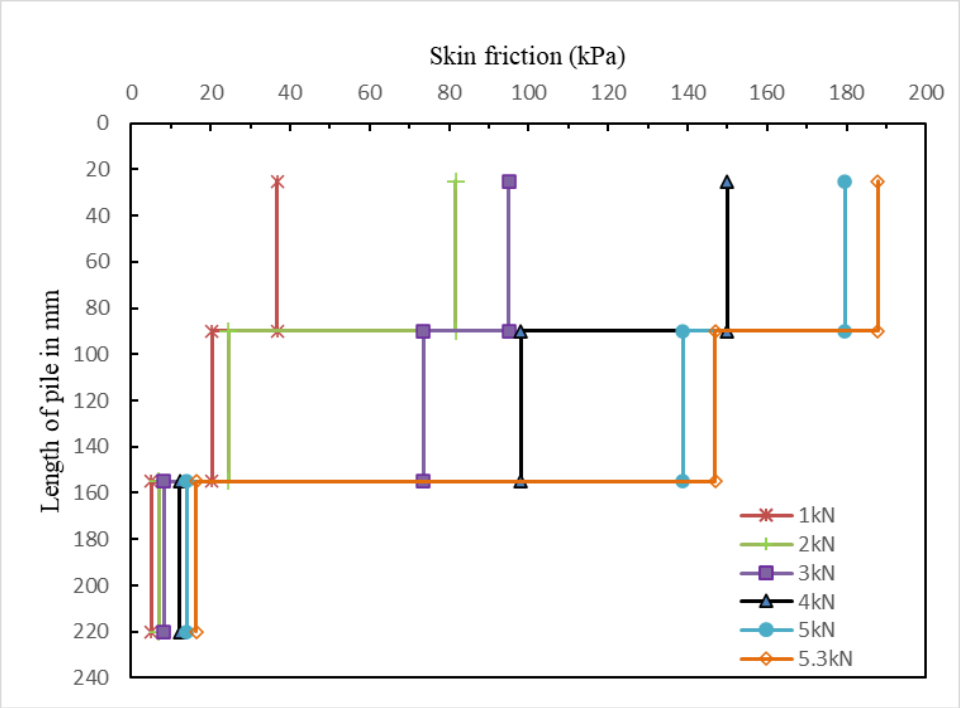
Fig. 11 (a) Axial load along pile with sponge, Ls/D = 4 (b) Axial load along pile without sponge, Ls/D = 4

5.8 CALCULATION OF UNIT SKIN FRICTION

Figure 12 depict the variation in unit skin friction throughout the height of piles with Ls/D ratios of 4 and 6. In general, it is discovered that the mobilized skin friction increases with increasing axial force until the failure loads. Utilizing axial loads measured at strain gauge locations, the mobilized skin friction was computed. Figure 12 show that the head of the pile is where the most skin friction is generated. Similar results from finite element analysis were found by [103] and [94]. Large pile settlement develops because to sponge compression, as seen in Figure 12, when end bearing resistance starts to contribute and skin friction at the pile tip decreases". This is in line with the finding of [95] and [104].



(a)



(b)

Fig. 12 (a) skin friction along pile with sponge, Ls/D = 4 (b) skin friction along pile without sponge, Ls/D = 4

5.9 NUMERICAL ANALYSIS OF AXIALLY LOADED PILE IN SAND

For the current numerical study, the programme PLAXIS 3D foundation is implemented. This is a finite element analysis programme used to analyze three-dimensional foundation structures deformations. Boreholes are used to identify the soil strata. Quadratic 10-node wedge components are used to simulate soil displacements and stress. For demonstrating findings, the PLAXIS postprocessor has 3D graphical elements. Specific displacement, stress, and strain data are taken from the output spreadsheets. For more precise results, the mesh is constructed with finer components. To improve precision, the soil cluster surrounding the pile is improved considerably. The pile is represented by a linear elastic material. The studies are divided into three stages. Maximum stresses in the soil are estimated in stage I, and pile placement is modeled in stage II. Axial load is transmitted to the pile top during stage III. Load is transferred in increments of up to 50 iterations in the finite element programme. The PLAXIS output software depicts settlement and stresses as tables and curves. The PLAXIS curves software generates load-settlement curves from the PLAXIS 3D study. Maximum settlement data are read from the load-settlement curves. The details of the input parameters for the

numerical model are tabulated in Table 1. Also, finite element modelling of sand and pile load are shown in Fig. 13 and 14 respectively.

Table 1 Properties of different parameter used in FE modelling.

Material	Sand	Sponge	Aluminum pile
Model	Mohr-coulomb	Linear elastic	Linear elastic
Unit weight, ɣ (kN/m^3)	17.7	1.3	27
Young's modulus, E (kPa)	20x10^3	3 x10^4	69x10^6
Cohesion, C (kPa)	5	-	-
Poisson's ratio μ	0.35	0.3	0.30
Friction angle, ϕ	32	-	-

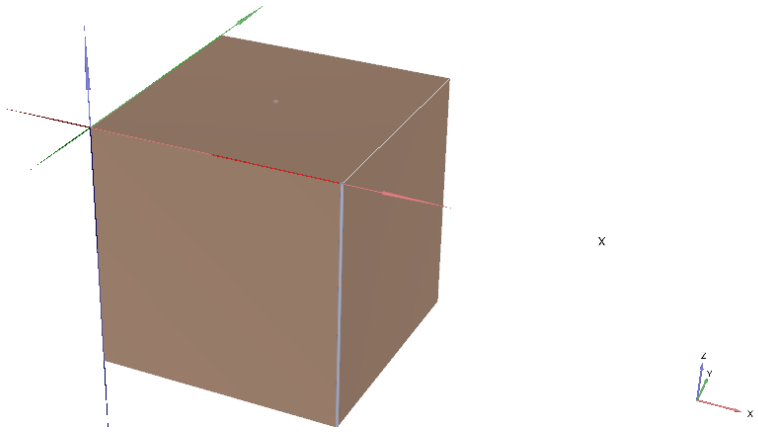


Fig. 13 FE model of sand layer

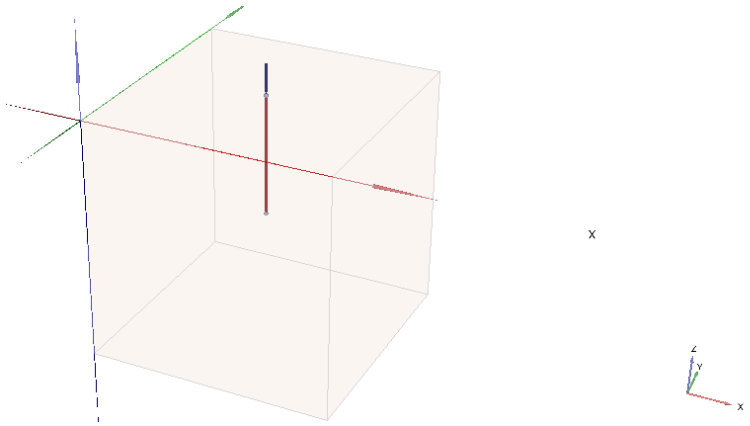


Fig. 14 FE model of the pile and load.

5.10 OUTPUT OF MODELLING

Output stage shows different stress location in the model. Its gives clear idea about critical stress zones in the model. It also shows vertical displacement, maximum stress and contours after loading. Fig. 15 shows vertical displacement after loading.

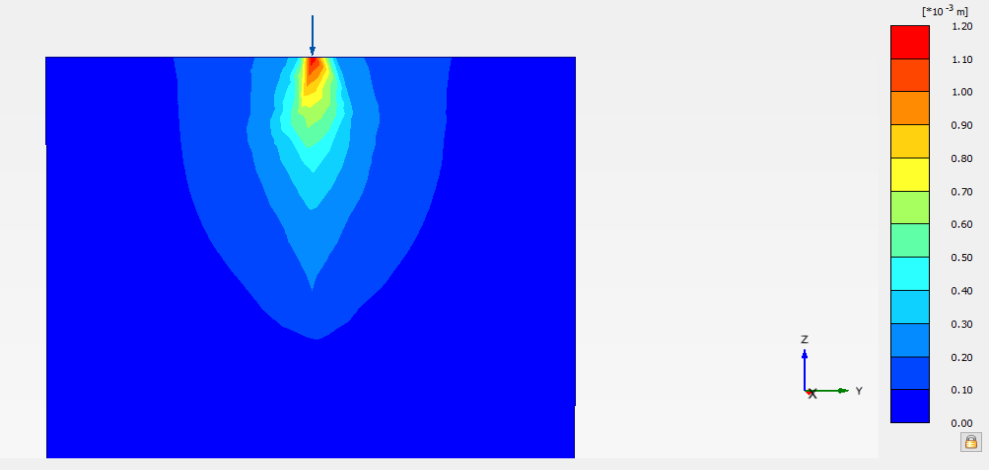


Fig. 15 FE model of total displacement in vertical direction

5.11 RESULTS

Figure 16 shows a comparative study of load-settlement graphs generated from laboratory tests and numerical modeling. It is obvious that the numerical simulation can adequately reflect the behavior of piles either with or without sponge at the pile tip. The numerically calculated settlement value for both piles is marginally lower than the value recorded in the experiments for a similar imposed load.

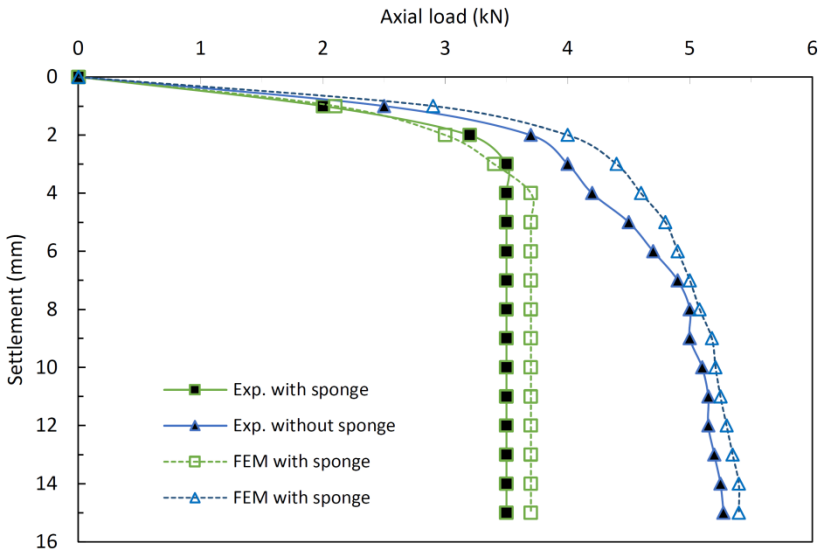
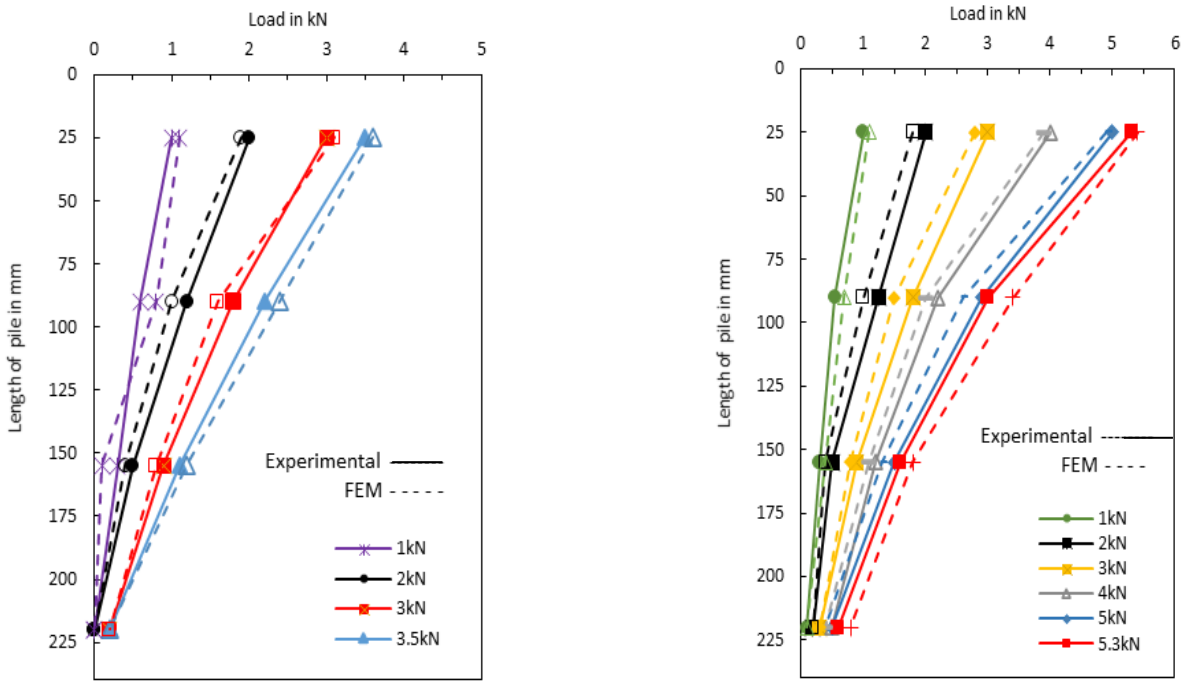


Fig. 16 Load-settlement curves obtained from experiments and numerical simulations for pile (4D) depth.

The measurement of axial load in the pile is depicted in Figure 17, including the computed results from numerical simulations. Larger load at pile top and decreasing with length. The load applied to the pile base is relatively low for the pile with sponge at the pile base, illustrating that pile capacity is predominantly produced by skin friction. After applying load, the soft zone of sponge beneath the pile is compressed, and the pile base resistance begins to contribution.

A number of FE simulations were used to calculate the load-settlement profiles for piles without and with sponge. Experimental results are correlated to numerical simulations. The load-settlement curves analyzed using the two approaches have typically good correlation. The displacement differences among laboratory and FEM findings are less than 5%.



(a) (b)

Fig. 17 Axial force of the pile obtained from experiments and numerical simulations: (a) Pile with sponge, and (b) Pile without sponge

6. CONCLUSION

In laboratory experimental studies determination of skin resistance of piles in socketed Pseudo-rock was examined through a point-by-point laboratory program. Experimental study on sand was carried out and ultimate skin friction was achieved and these results found to compare very well with the Numerical methods. The following are the major outcomes gained from the current studies.

1. A new empirical equation is developed by using experimental data of skin friction in the socketed length of the pile and uniaxial compressive strength of pseudo-rock. equation (4.9) can be utilized to calculate the maximum skin friction values of socketed piles in rock. The skin friction values calculated using the suggested empirical equation are roughly 5 to 12 percent lower than the skin friction values calculated using the relations suggested by [24]. The values of skin friction derived using the recommended equation and the equations presented by [8] and [24] match fairly well.
2. The deployment of unit skin friction is significantly influenced by the rock's strength. Referring to the testing data, skin friction values improve as pseudo-compressive rock's strength (qu) rises.

3. The elasticity modulus and UCS of pseudo-rock were observed to reduce as the amount of PoP is increased. These is because, as sand content is increased, and cement content is decreased, because of which bond between sand particles are getting reduced. as a result, pseudo-rock strengths decreased as pop content increased.
4. When piles were under compressive load, a high rate of skin friction was generated towards the top, and it was later shown that this rate was decreasing with pile length.
5. Using a specifically design and developed laboratory set-up for examining piles, investigations on the skin friction of socketed piles are conducted experimentally. The study has clearly demonstrated that the optimum socket depth can be up to five times the pile diameter. The improvement in pile capacity is found to be minimal beyond this length.
6. Pile with sponge at the toe is found to have low bearing capacity as compared to pile without sponge.
7. Once the piles were associated to compressive loads, a larger rate of skin friction was observed to be mobilized towards the top of pile, which then decreased with pile length.
8. A number of FE simulations were used to calculate the load-settlement profiles for piles without and with sponge. Experimental results are correlated to numerical simulations. The load-settlement curves analyzed using the two approaches have typically good correlation. The displacement differences among laboratory and FEM findings are less than 5%.

7. FUTURE SCOPE

1. Quantifying the roughness and modeling of roughness between pile surface and the rock socket.
2. fractures and discontinuity may be considered and modelled for the determination of Skin friction
3. Numerical analysis can be carried out on group of piles.

8. PUBLICATIONS

following are the details of the publications of the research papers and IPR.

1. Vedprakash C. Marlapalle and Ramachandra Hegde, "Model studies on effect of pseudo- rock-socket strength on resistance of friction-only piles," Engineering Science and Technology, an International Journal, Elsevier, Volume 34, October 2022, 101089.

(Indexed: Scopus, web of science, SCIE).
https://doi.org/10.1016/j.jestch.2021.101089

2. Vedprakash C. Marlapalle and Ramachandra Hegde, "Experimental and empirical study on piles socketed into rock in Navi Mumbai", Soil Mechanics and Foundation Engineering, Springer. **(Indexed: Scopus) (Accepted)**

3. Vedprakash C. Marlapalle and Ramachandra Hegde, "Experimental study on socketed pile in soft-rock", Engineering, Technology & Applied Science Research. **(Indexed: web of science) (Accepted)**

4. Vedprakash C. Marlapalle, Ramachandra Hegde, Rohan Dasgupta, Wasim Shaikh "Experimental Investigation on Piled Raft Foundation" International Journal for Science and Advance Research in Technology, Volume 3, Issue 6, Pp 666 – 670, June 2017. **(Indexed: UGC)**

5. Vedprakash C. Marlapalle, Ramachandra Hegde, Rohan Dasgupta, Wasim Shaikh "Experimental Study on Behaviour of Piled -Raft Foundation - Critical Review" International Journal for Science and Advance Research in Technology, Volume 3, Issue 6, Pp 671 – 673, June 2017. **(Indexed: UGC)**

IPR filled details.

1. Patent Title: Compression Testing Equipment for Testing Model Piles.
   Applicant/Inventor: Vedprakash Maralapalle, Dr. Ramchandra Hegde
   Application No: 201921027374
   Date of filing: 09/07/2019
   Publication Date:22/05/2020
   Application Status: Under examination
   https://search.ipindia.gov.in/IPOJournal/Journal/ViewJournal

2. Patent Title: Multi-Purpose Testing Equipment for Testing Model Piles.
   Applicant/Inventor: Vedprakash Maralapalle, Dr. Ramchandra Hegde
   Application No: 201921027375

Date of filing: 09/07/2019

Publication Date: 22/05/2020

Application Status: Under examination

3. Design Registration (Design Patent): Compression Testing Equipment for Testing Model Piles.

   Applicant/Inventor: Vedprakash Maralapalle, Dr. Ramchandra Hegde

   Application No: 320703-001.

   Date of filing: 14/08/2019

   Application Status: Under examination

4. Design Registration (Design Patent): Equipment for Testing Model Piles with Inclined Loading.

   Applicant/Inventor: Vedprakash Maralapalle, Dr. Ramchandra Hegde

   Application No: 320704-001.

   Date of filing: 14/08/2019

   Application Status: Accepted

   Journal No is 42/2020 and Journal Date is 16/10/2020

   https://search.ipindia.gov.in/IPOJournal/Journal/ViewJournal

5. Design Registration (Design Patent): Equipment for Testing Model Piles with Lateral Loading.

   Applicant/Inventor: Vedprakash Maralapalle, Dr. Ramchandra Hegde

   Application No: 320705-001.

   Date of filing: 14/08/2019

   Application Status: Under examination

6. Design Registration (Design Patent): Equipment for Testing Model Piles with Tensile Loading.

   Applicant/Inventor: Vedprakash Maralapalle, Dr. Ramchandra Hegde

   Application No: 320706-001.

   Date of filing: 14/08/2019

   Application Status: Accepted

   Journal No is 27/2020 and Journal Date is 03/07/2020

   https://search.ipindia.gov.in/IPOJournal/Journal/ViewJournal

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